Chapter 8
Loads

8-1. General

The chapter contains discussion and guidance on loads that can normally be expected to be imparted to a lock or any appurtenant structure. This discussion includes live loads due to hydraulic forces, impact, seismic and ice forces, earth pressures, and thermal stresses. Dead loads include weight of the structure and equipment.

8-2. Hydrostatic

For hydrostatic loads, the damming action of a lock must be effected at both the upper and lower gates. The water pressure against the face of the lock walls and the base slab is variable and depends on the waterway stages that prevail at a particular time or on other water conditions which may produce higher pressures. For those elements of a lock that are not required to resist lateral earth thrusts in conjunction with water pressures, the maximum pressures are easily determined. However, most lock locations have backfill adjacent to least one wall.

a. Horizontal water pressures. No definite rule can be followed in determining the level of the groundwater in the backfill adjacent to the back face of a lock wall. However, it is well established that the saturation level varies between the upper and lower pool elevations, and the degree of variation depends on the physical characteristics of the backfill material and the permeability of the foundation. The location of the saturation line should be determined analytically, based upon laboratory tests of the dry and wet characteristics of the soils, the extent of compaction to be used, and the effect of local climatic conditions.

(1) A majority of the navigation locks in the United States are located in natural waterways where the backfill material has granular characteristics. This material has a tendency to drain and to become saturated with an approximately straight-line variation between pool elevations. For projects with fairly stable pool levels, these assumptions should be sufficiently accurate to give satisfactory results. However, varying pool levels and use of impervious backfill material will probably cause considerable departure from straight-line variation. For lock installations with a lower pool subject to greater fluctuations than the upper pool, a lower pool stage that is exceeded not more than a small part of the time should be selected from the stage duration curves. In this case, the saturation line can be constructed between this lower pool level and the normal upper pool level, and the height of the groundwater table can be determined accordingly for that portion of wall under consideration. The extent of saturated earth must be established with a reasonable degree of accuracy in order to accurately represent horizontal force due to earth and water pressures and uplift.

(2) In addition to the usual stabilized groundwater levels caused by normal discharges, extreme loading conditions due to raised saturation levels must also be investigated. These conditions include the effect caused by locally heavy rains without an accompanying rise in the pool stages and by flood discharges that overtop the walls which cause the earth to become saturated throughout. Following flood discharges, the pool levels often approach their normal levels more rapidly than the fill material can drain. Although these increased loads are serious and should be investigated, the stability requirements are usually relaxed because of their short duration and the infrequent occurrence of such increased loads. For this condition, the assumption is often made to raise the saturation line above its normal location to one-half the distance to the top of the fill. The remainder of the fill is considered as moist earth. An effective method of controlling the saturation line is to provide a collector pipe surrounded by a properly designed filter material at some convenient level above the lower pool. This method can consequently reduce the water pressure against the wall in a free-draining material. To create an effective system, an impervious zone must be placed at the upper end of the lock wall extending to the natural bankline or to some other adjacent structure. This system will form an effective damming surface and retard flow of the upper pool groundwater to the lower pool through the fill material. This type of design not only effectively lowers the saturation line for the normal operating conditions but also will facilitate drainage of the backfill immediately after the extreme loading conditions.

b. Uplift. The problem of uplift for lock walls is complicated by fluctuating water levels within a lock chamber. The rate of change of uplift as the chamber is filled or emptied is not known.

(1) Rock foundations.

(a) Upper pool in lock chamber. For the river wall, uplift will vary from 100 percent of lower pool at the riverward face to 100 percent of upper pool at the chamber face. For the land wall, uplift will vary from 100 percent of the head represented by the saturation level in the backfill at the landward face to 100 percent of upper pool head at the chamber face.

(b) Lower pool in lock chamber. For the river wall, a uniform uplift of 100 percent of lower pool head will exist under the entire base. For the land wall, uplift will vary from 100 percent of the head represented by the saturation level in the backfill at the landward face to 100 percent of lower pool head at the chamber face.

(c) Maintenance. For the river wall, uplift will vary from 100 percent of lower pool head at the riverward face to a head represented by the pumped down level at the chamber face. For the land wall, uplift will vary from the saturation level at the landward face to the pumped down level at the chamber face.

(d) Construction. Uplift acting on the base of any monoliths within the cofferdam is assumed to be zero.

(e) Drainage. In cases where adequate drainage (relieving to tailwater) is provided near the chamber face, total uplift may be reduced for the condition of upper pool in the lock chamber. For river walls, uplift will vary from 100 percent of tailwater plus 50 percent of the difference between headwater and tailwater at the chamber face to 100 percent of tailwater at the river face. For land walls, use the saturation line instead of tailwater. Probably the most effective land wall drainage is that provided in the backfill to reduce the saturation level.

(2) Soil and pile foundations. Monoliths on soil or pile foundations usually have cutoff walls and sometimes have drainage systems. At one face of the monolith, uplift should be the full headwater pressure from the face of the wall to the cutoff. At the other face, uplift equals the full tailwater pressure (or the saturation head in the backfill). Uplift pressures between these points should be determined by evaluations of cutoff and drain effectiveness and soil permeability. Cutoffs and drains will provide at least a 50 percent reduction in uplift, similar to rock foundations. Under ideal conditions, cutoffs and drains can be considered 100 percent effective in reducing uplift pressures.

(3) Seepage paths. The uplift under U-frame locks is complicated by alternative seepage paths along and perpendicular to the lock axis. The permeability of the foundation soils, as well as the existence of sheet pile cutoff walls and foundation drains, affect the variation of the uplift pressure. Close coordination with geotechnical engineers is needed to determine the uplift pressure for each lock monolith. All combinations of operating and maintenance conditions should be analyzed to determine the most critical condition.

(4) Uplift. Except for earthquake loading, any portion of the base, not in compression, will be assumed to sustain a uniform uplift equivalent to 100 percent of the adjacent pool or saturation level. Uplift for loading which includes earthquake will be assumed to be equal to that for the same loading without earthquake.

(5) Internal stresses. Because minor movements of gate sills affect the gate operations, all sill blocks should be analyzed for stability and for internal stresses resulting from maximum differential heads. Uplift on the base and hydrostatic internal pressures will be assumed to vary from 100 percent of high-water pressure to 100 percent of low-water pressure. Any portion of the base or horizontal internal plane not in compression should have a uniform uplift of 100 percent of the high-water head acting upon it.

8-3. Earth Loads

In lock wall design, careful investigation of available backfill materials and methods of backfilling is of primary importance.

a. Horizontal earth pressures. Generally, “at-rest” pressures should be used for gravity sections on rock foundations. Few rock foundations yield enough to develop active pressure values. Values for these pressures should be determined for the various conditions of the backfill (drained, saturated, or submerged) by accepted soil analysis methods. Horizontal earth pressures below assumed saturated levels will be computed at the same horizontal earth pressure coefficient used for the drained or saturated condition. At-rest pressures should be used for other foundation types unless sufficient movement is expected to result in active soil pressures.

b. Silt. Model studies can only indicate tendencies for location of silt accumulations. If the model studies indicate tendencies for silt buildup, the most conservative assumptions for depth of silt should be used, and corresponding vertical and horizontal loads should be used in the analysis.

c. Vertical shear (downdrag). Walls (lock walls, approach walls, miscellaneous walls) that retain fill should consider vertical shear in the stability analysis and in the slab design in the case of a U-frame or W-frame. The vertical shear is a result of a change in the state of stress in the soil backfill as vertical loading changes. These changes occur during initial construction (backfill is consolidating) or during unwatering of a U-frame or W-frame (the lock wall is tending to move up with respect to the stationary backfill). The computational procedure for computing the vertical shear is to use a vertical shear coefficient K_v applied to the effective vertical stress of the soil. The vertical shear is then applied at a plane extended at the outermost extremity of the wall.

8-4. Earthquake or Seismic

Two general approaches to determining seismic forces include the seismic coefficient method and a dynamic analysis procedure. The seismic coefficient method (also known as the pseudostatic method) should be used only as a preliminary means to determine the location of resultant and sliding stability of lock monoliths, or to analyze gravity lock walls at horizontal sections to determine if the horizontal section is in 100 percent compression. If the seismic loads computed by the seismic coefficient method indicate a problem, a more rigorous dynamic analysis should be performed as described in Chapter 10. Seismic coefficients are based on the seismic zones provided in ER 1110-2-1806. Details of this procedure are contained in EM 1110-2-2200.

8-5. Impact

Tows operating on inland waterways can lose control and collide with lock walls. The design of navigation lock approaches is influenced to a great extent by impact loads from collisions. The magnitude of the impact forces depends on the mass including the hydrodynamic added mass of the barge tow, the approach velocity, the approach angle, the barge tow moment of inertia, damage sustained by the barge structure, friction between the barge and wall, and the flexibility of the approach wall. An analytic method which can be used to approximate the maximum impact forces on structures located in navigable waterways is presented in ETL 1110-2-338.

8-6. Hawser Loads

Hawser loads are generated by barges checking their movement into a lock by tying-off to a line hook or check post, or by loads imparted from movement perpendicular to a lock wall as a result of turbulence caused by filling or emptying the lock chamber. Since the filling and emptying system is generally designed so that the hawser pull is limited to 10 kips, the controlling load will be the checking hawser pull.

a. Lock wall. The lock wall concrete should be designed for the parting strength of the strongest hawser anticipated to be used in the navigation system. Currently, the recommended hawser pull is 160 kips. The location of the load should be either 5 ft above the waterline or 1 ft above the lock wall (whichever is closest to the waterline). The angle of the load application should be consistent with a barge entering a lock and checking its forward movement by tying-off to a line hook or check post. Unless demonstrated otherwise, an angle of application of 30 deg with the wall should be used; therefore, the perpendicular component is 80 kips.

b. Line hooks, check posts, and floating mooring bitts. A hawser pull of 160 kips should be used for the design of line hooks, check posts, and floating mooring bitts and their anchorages.

8-7. Ice and Debris

Usually lock designs do not include the effects of ice loads on lock walls. However, approach walls, particularly those located in the upper approach, are sometimes subjected to moving ice and floating debris, and the wall designs should account for these effects. For projects where ice conditions are severe and where the ice sheet is short or can be retrained or wedged between structures, its magnitude should be estimated with consideration given to the locality and available records of ice conditions. A unit pressure of not more than 5,000 lb/sq ft applied to the contact surface of the structure at normal pool level is recommended. In the contiguous United States, the ice thickness assumed for design normally will not exceed 2 ft resulting in a design load of 10,000 lb/ft. However, a minimum load of 5,000 lb/ft is recommended to account for ice and debris. Further discussion on types of ice-structure interaction and methods for computing ice forces is provided in EM 1110-2-1612.

8-8. Wave Pressure

While wave pressures are of more importance in their effect upon gates and appurtenances, they may, in some instances, have an appreciable effect upon the lock or approach walls. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors. Information relating to waves and wave pressures are presented in SPM (1984).

8-9. Wind Loads

Wind loads usually need not be included in the wall analysis, except where major portions of the walls are not backfilled, or where projections such as bridge piers and spans or control houses extend above their tops. If the design includes wind forces, the forces should be placed in the most unfavorable direction and should be assumed at 30 lb/sq ft unless past records indicate that other assumptions are warranted. TM 5-809-1 contains additional guidance on wind loading.

8-10. Gate Loads and Bulkhead Loads

a. Miter gates.

(1) Figure 8-1 shows the gate orientation and methods of determining the total thrust as applied to the wall for normal operating conditions.

(2) For horizontally framed gates, all of the water loads cause gate thrust which is transmitted through the girders and quoin blocking into the gate monolith. None of the load is carried into the sill beam. For vertically framed miter gates, part of the gate water load (R_1) is carried by a top girder of the gate and transmitted by it to the wall; the other part (R_2) is carried directly to the sill. The thrust of a vertically framed gate applied to the wall at the center line of the top girder is the same as for a horizontally framed gate except that P is substituted by R_1.

(3) The gate dead loads may have a more severe overturning effect on the gravity lock walls when there is no water load on the gates. To obtain the maximum overturning effect, the gate leaves are considered to be swinging free in approximately the mitered position.

b. Other gates and bulkheads. Gate reactions from dead loads and hydrostatic loads must be considered during stability analysis and detailed design of all gate monoliths.

8-11. Crane, Equipment, Machinery, and Appurtenant Items

Loads due to machinery and appurtenant items are normally small but should be included when present. These loads may result from items such as miter gate operating machinery, tainter valve operating machinery, emergency bulkhead lowering carriage machinery, emergency cranes for placement of stoplogs, or stoplogs stacked in either their storage location or the stoplog slots in the lock walls. These loads should be applied as point loads at the appropriate locations on the lock walls.

8-12. Cofferdam Tie-in Load

Cofferdam tie-in loads are caused by either cellular or embankment cofferdams constructed for a phase subsequent to the lock construction by using the lock structure as a portion of the dewatering cofferdam. These loads must be accounted for in the design, but they are normally considered an unusual condition.

8-13. Sheet Pile Cutoff Loads

Differential water pressure may exist on opposite sides of sheet pile cutoffs attached to the bottom of the lock monolith (see Plate 41). Past analyses have shown that the load on the monolith from this condition is small and can usually be neglected.

8-14. Monolith Joint Loads

Loadings in the upstream-downstream direction should be evaluated based on the critical condition of waterstops being ruptured or intact to give the greatest driving load. Typical waterstop details are shown on Plate 42.

8-15. Superstructure and Bridge Loads

Superstructure loads are defined as loads imparted to the lock walls as a result of structures such as the control house, electrical control structure, and any other enclosures needed for lock operation. Bridge loads are imparted to the lock wall as a result of piers that support a service, access, pedestrian, or highway bridge. Superstructure or bridge loads should be considered in the stability computations as well as for determining localized stresses in the concrete lock wall.

8-16. Temperature

a. A major concern in concrete lock construction is how to control cracking that results from temperature change. During the hydration process, the temperature rises because of the hydration of cement. The edges of the monolith release heat faster than the interior; thus, the core will be in compression and the edges in tension. When the strength of the concrete is exceeded, cracks will appear on the surface. When the monolith starts cooling, the contraction of the concrete is restrained by the foundation or concrete layers that have already cooled and

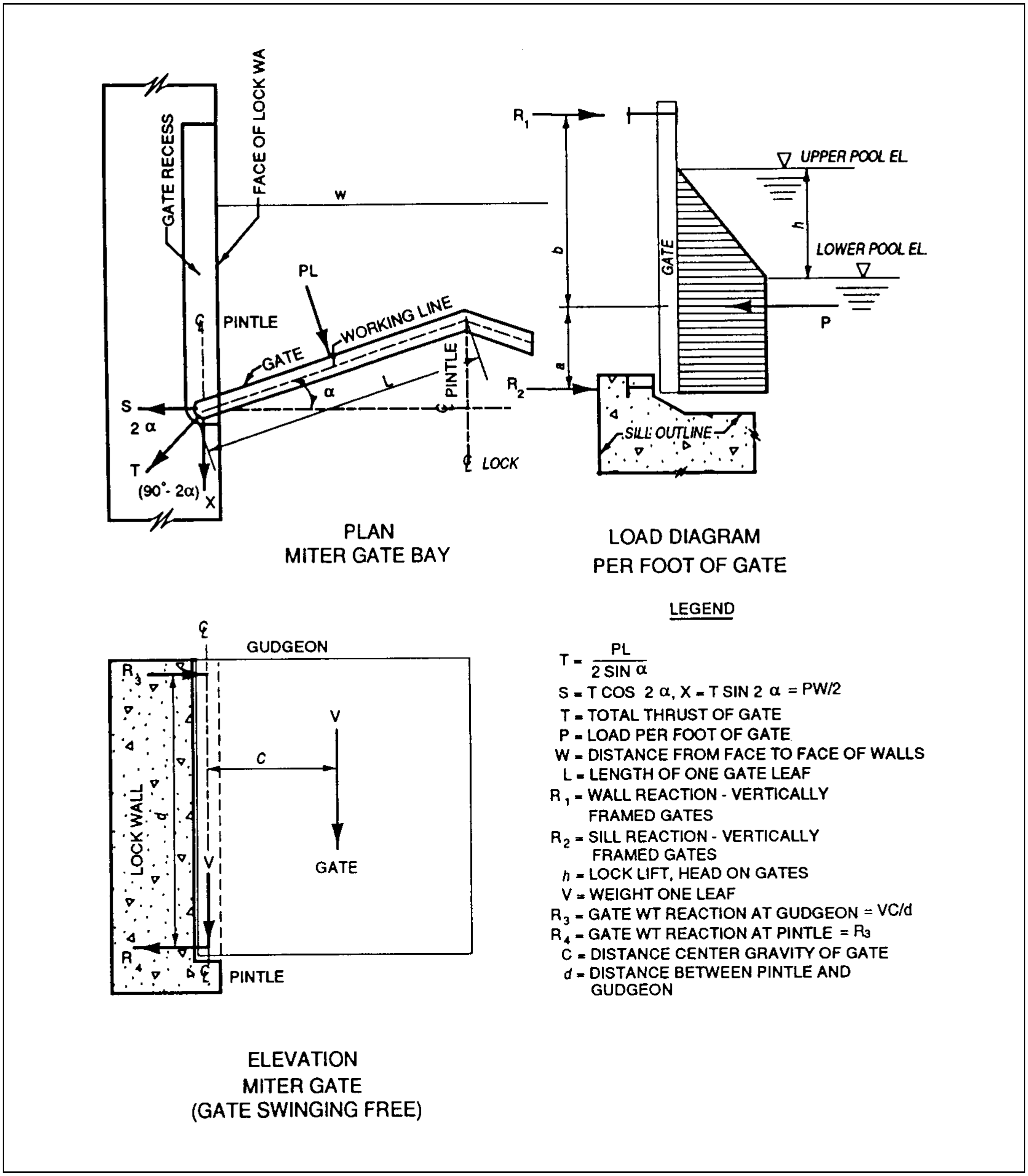

Figure 8-1. Miter gate, wall, and sill loads

hardened. Again, if this tensile strain exceeds the capacity of the concrete, cracks will propagate completely through the monolith. Cracking affects the watertightness, durability, appearance, and stresses throughout the structure and may lead to undesirable crack propagation which impairs structural safety.

b. Various techniques have been developed to reduce the potential for temperature cracking in mass concrete (ACI 224 R-80, ETL 1110-2-365). Besides contraction joints, these methods include temperature control measures during construction, cements and pozzolan for limiting heat of hydration, and mix designs with increased tensile strain capacity.

8-17. Hydrodynamic Loads

Hydrodynamic loads on lock walls may exist when the lock is constructed adjacent to the navigation dam. In this case, the outside of the lock is exposed to the turbulence of the water being discharged from the dam. The high-velocity flows and fluctuating water surface due to the dissipation of energy in the hydraulic jump cause low-frequency pressure fluctuations that should be considered in the design of the lock walls. The frequency, magnitude, and areal extent of the pressures is site-specific and can be determined only by close coordination with hydraulic engineers.

8-18. Cyclic Loads

The nature of loads is cyclic due to the purpose of the project (raise and lower the water level in the lock chamber many times daily). Any components of the lock which are sensitive to cyclic loading should include the effects of these loads. Examples of components sensitive to cyclic loading include pile foundation design and reduction of design stress in steel and welded connections.

8-19. Internal Hydrostatic Pore Pressure in Concrete

Hydrostatic pressures within the body of the lock wall should be assumed to vary from 100 percent of tailwater at the low-water face to 100 percent of tailwater plus 50 percent of the difference between headwater and tailwater at the high-water face. The resulting force acting on a cross section will reduce the axial compression (or increase the axial tension) and should be considered in the stress analysis when it is conservative to do so.